Report No. UT-99.15

FORCED VIBRATION TESTING OF THE I-15 SOUTH TEMPLE BRIDGE

PHASE 1 FINAL REPORT

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Submitted By:
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UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

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16. Abstract

The objective of this research program is to investigate the potential for using system identification as a non-destructive evaluation technique. This research examines the feasibility of performing system identification on a large, multi-degree of freedom structure and a simple, single span structure. The testing consisted of performing sine sweeps over a range of excitation frequencies, with the excitation induced in the horizontal direction by an eccentric mass shaker. The response of the two bridge structures was recorded with accelerometers. The simple span structure was tested in seven condition states that included post-damage testing.

In the case of the nine-span bridge the lowest five response modes and frequencies were determined, demonstrating that system identification of large bridge structures is possible. For the simple span structure the lowest three mode shapes and frequencies were determined for each condition state. The change in the natural frequencies for each condition state demonstrated the new condition of the bridge, whether it was post-damage or post-repair. This indicates that system identification (modal analysis) has potential as a non-destructive evaluation method for determining structural integrity.

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I-15 SOUTH TEMPLE BRIDGE

A Research Report Submitted to

The Utah Department of Transportation

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Executive Summary

The objective of this research is to investigate the potential for using system identification, the determination of a structural system's response characteristics through dynamic testing procedures, as a non-destructive evaluation technique to assess damage. This project consisted of two phases. The first examined the feasibility of performing a system identification for a large, multi-degree of freedom (MDOF) structure in the form of a multi-span bridge. The second phase consisted of a forced vibration study of a simple bridge span, the remnant of the multi-span bridge after demolition, in various states of damage and repair. The bridge selected for this research, made available through the I-15 reconstruction, was the northbound I-15 bridge over South Temple Street. This was a nine-span, skewed bridge with a considerable amount of open area underneath that making the physical testing of the bridge possible.

Previous studies on system identification have focused on single-degree of freedom (SDOF) systems and reveal the benefits of, and methods for, system identification. System identification becomes much more complicated when applied to large, MDOF systems. This project reveals the difficulties in testing large, in-situ structures and the modifications of typical SDOF testing methods required to accommodate large MDOF structures.

The results of this research in terms of frequencies and mode shapes for the nine-span bridge are shown as well as the changes in the frequencies and mode shapes for the simple span bridge as it went through various states of damage and repair. This study shows that system identification of large structures is possible and gives a preliminary indication that system identification has potential as a non-destructive evaluation technique for the determination of structural damage.

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I. Introduction

Determining a structural system's physical characteristics through dynamic testing procedures is known as system identification and has been explored over the past two decades [1,2]. Prior to system identification most field tests were static in nature and could not reveal many of the properties of a given structure [3]. Current research in the area of system identification is focusing on its capabilities and potential application as a method of structural damage and integrity assessment.

The application of system identification in this way is based on the fact that dynamic response is sensitive to the integrity of a structure. Local or global damage can cause a reduction in stiffness and a decrease in the free energy stored within a structural system [4]. Changes in stiffness, mass, and damping will lead to changes in the natural frequencies, mode shapes and modal damping of a structure. It is assumed that in most cases of structural damage the change in mass and damping will be minor in compared to the change in stiffness of a structure. Therefore, any change in the dynamic response of a structure will be due to a change in stiffness, which would be the result of structural damage.

Using structural identification as a non-destructive evaluation technique for damage detection requires an initial structural identification of a structure, the "before" snapshot, where the dynamic characteristics are determined for the undamaged structure. Then after a natural disaster (earthquake, flood, etc.) or a collision the structure undergoes a second system identification, the "after" snapshot. If the structure has been damaged, then the dynamic characteristics will have changed, due primarily to the change in stiffness. This change in stiffness will be detected by the structural identification indicating damage to the structure.

Methods of structural excitation that may be used for system identification include ambient, impact and forced vibration techniques. This type of test has been conducted on buildings, earth and arch dams, fluid-filled storage tanks, water-intake towers and bridges [1,2]. These tests provide some information as to the testing techniques required for a full scale multi-degree of freedom structure. However, much of the system identification research has taken place on models [2,5]. For damage assessment and structural integrity analysis through system identification to be accepted by practicing engineers, there must be more studies performed on the feasibility, practicality and validity of applying these principles to full scale structures.

The purpose of this research was to investigate the feasibility of using system identification of large structures as a practical method of non-destructive damage detection. Ultimately it is anticipated that system identification might be implemented as a routine damage detection method. Bridges within the state would be tested and their dynamic characteristics logged in a database with subsequent tests performed every few years or immediately after a catastrophic event so that the condition of these bridges might be assessed based on the changes in their dynamic characteristics.

This project focused on the northbound I-15 bridge over South Temple Street in Salt Lake City. In March of 1998, the Utah Department of Transportation (UDOT) and Wasatch Constructors made this structure available for testing, prior to its demolition. Initially the focus was on the entire nine-span structure and conducting a dynamic study of this structure using forced vibration. Then after the demolition of most of the South Temple overpass, leaving only a simple span, the research focused on detecting through system identification (again using forced vibration) the damage state of the smaller structure as it went through various phases of damage and repair. The damage and repair of the structure was done by researchers at the University of Utah [6].

II. Testing of Nine-Span Bridge

Bridge Description

The bridge tested was the complete northbound overpass structure crossing South Temple Street in Salt Lake City. This bridge had nine simple spans of composite construction with a total length of 188.76 meters (619 feet) and a width of 18.28 meters (60 feet). The simple spans consisted of reinforced concrete bents, abutments and deck that were used in conjunction with steel plate girders. The spans were not of equal length, varying from 15.32 to 28.29 meters (50.25 to 92.83 feet) in length, see Figure 1.

Testing Equipment

The dynamic excitation of the bridge was provided by an AFB Engineered Test Systems Model 4600A eccentric mass shaker capable of providing a sinusoidal forcing function in any horizontal direction. The amplitude of the force induced by the shaker can be up to 89 kN (20,000 lbf) at up to 20 Hz. The magnitude of the excitation is a function of the frequency of vibration. The shaker was positioned on the transverse centerline of bridge span number six about 6 meters (19.68 feet) to the west of the bridge centerline. The eccentricity of the machine was set to provide full harmonic excitation in the transverse direction, perpendicular to the centerline of the bridge, see Figure 1. The machine was not moved during the testing of the nine-span bridge or the single span.

The dynamic response of the bridge was measured utilizing Kinemetrics FBA-11 uniaxial 1g and 0.25 g accelerometers. The array of accelerometers was such that there was an accelerometer placed in the center of every span, oriented to measure the response perpendicular to the centerline of the bridge (Figure 1). This array utilized nine of the available ten accelerometers. The objective of placing the instruments on the bridge in this manner was to capture the first four transverse modes of the bridge.

The data acquisition system consisted of two main components. The accelerometers and shaker were connected to a Kinematics VSS-3000 vibration survey box. This box contains an analog to digital converter that took the analog signals provided by the accelerometers and shaker and converted them to digital signals that can be stored in virtual space. This converter box is equipped to handle up to 16 input channels at one time and can accommodate sampling rates of up to 100 kHz. The second component consisted of a desktop computer that utilized data acquisition software to control the sampling rate and operation of the converter box. Once the signals were converted to digital signals they were sent to the computer where they were stored in ASCII format in preparation for later data processing and analysis.

Testing Procedure

After installing the eccentric mass shaker and accelerometers on the bridge deck and setting up the data acquisition system the bridge was tested. The test regimen consisted of subjecting the bridge to excitation using the eccentric mass shaker which was set at increasingly higher frequencies. This is know as a frequency sweep, the shaker is set to an excitation frequency and allowed time to reach that frequency. After the shaker has settled in at the set frequency the bridge is allowed to reach a steady state response which is then measured using the accelerometers. Then the shaker was adjusted to a new, higher frequency for which the bridge response was again recorded. This procedure was repeated until the testing of the bridge was complete.

An initial computer model of the bridge was developed to approximately determine the natural frequencies and mode shapes of the nine-span bridge. This model showed the first four transverse frequencies to be less than 9 Hz. This allowed the frequency sweep to be capped at 11 Hz and helped to determine that the frequency sweep should be stepped up in 0.05 Hz increments to ensure that no modes would be missed.

When collecting dynamic response data there are two options: to collect data at low sampling rates over a long period of time or at high sampling rates over short periods of time. Either way there must be a minimum number of data points available to clearly and accurately define a response. Due to the large number of frequencies at which this bridge was to be tested a sampling rate of 100 Hz for a period of 20 seconds was chosen in an attempt to both capture the response of the bridge at each excitation frequency and limit the total testing time to a reasonable amount. A sampling rate of 100 Hz was nine times higher than the upper limit test frequency of 11 Hz which under normal circumstances is more than sufficient.

At 5.5 Hz of excitation the amount of weight used in the mass shaker had to be reduced to prevent damage to the machine. This reduction in weight resulted in an initial reduction of the excitation force and, therefore, a reduction in the bridge response. However, the natural frequencies and mode shapes that occurred in the bridge after the reduction in the weights were still detectable, even at lower levels of excitation.

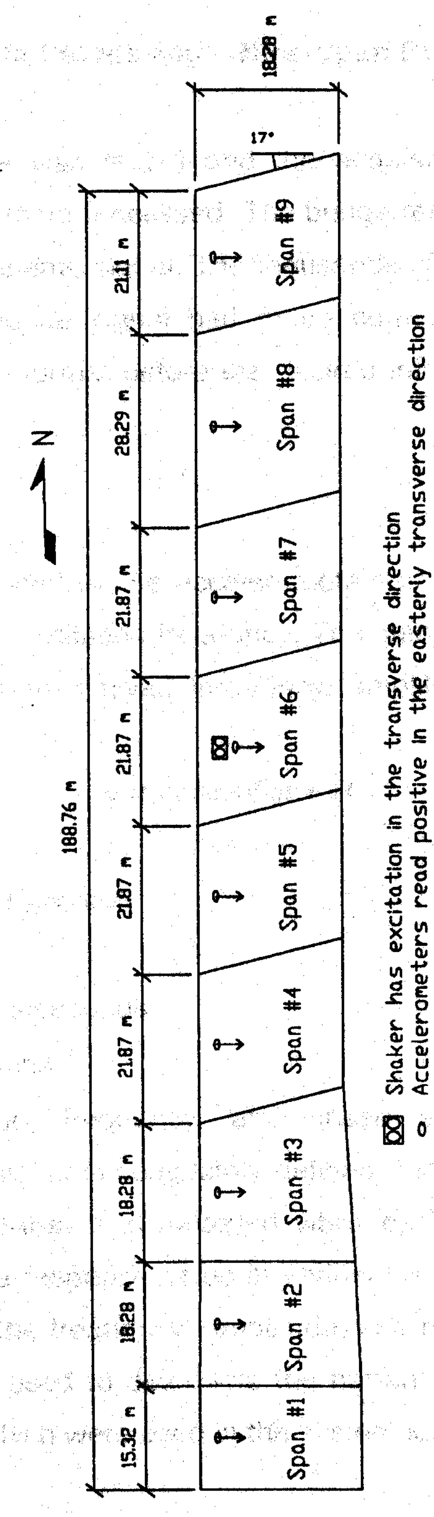


Figure 1. Plan of Nine-Span I-15 Overpass at South Temple

III. Data Processing – Nine-Span Bridge Test

Once the bridge was tested and the acquisition of its response was recorded, the data had to be processed. The bridge response data was recorded in the form of a discrete-time signal. The thousands of data points that made up any given bridge response signal had to be correlated to one another and converted into a usable format before the desired information could be gleaned from them.

Signal

The signal produced by the accelerometers is the steady state response of the bridge at a given excitation frequency. The steady state response is in the form of a sine wave with a given frequency, amplitude, and phase angle as shown in Equation 1.

$$y(t) = A\sin(\omega t + \phi) \qquad (1)$$

Where:

$y(t)$ = response at time t seconds

A = amplitude

ω = frequency in radians/seconds

ϕ = phase angle in radians.

If the amplitude, frequency, and phase angle of the signals are determined, then the signal is completely defined. Figure 2 gives an example of an acceleration signal that was recorded while exciting the nine-span bridge. Once the steady state response of each channel at each excitation frequency was determined then the frequency, amplitude, and relative phase angles of the response signal were used to determine the natural frequencies, mode shapes and modal damping which were used in the system identification of the bridge.

Excitation Frequency

The excitation frequency is determined by a signal coming from the shaker into the data acquisition system. The signal from the shaker is a pulse generated each time the shaker is at full force in the transverse east direction. Figure 3 gives an example of the type of counting signal the shaker produces. The signal has the main pulse, which indicates the time of maximum forcing in the easterly transverse direction but also has many other small pulses due to noise in the signal. A filter could have been applied to rid the signal of the noise, however, a filter that didn't destroy the integrity of the sawtooth curve was cumbersome to generate and not really necessary. An algorithm that could determine the shaker signal's frequency with the noise present was used.

By changing the signal from the time domain to the frequency domain, the frequency of the shaker signal can easily be picked out. The process used to determine the predominant frequencies of the signal's discrete-time Fourier transform is called the power spectral density (PSD) of the signal [5]. Once this conversion has taken place then the dominant excitation response frequency may be determined. Figure 4 represents the PSD of the excitation signal shown in Figure 3. It can be seen that the point of greatest magnitude occurs at 1.74 Hz which is the excitation frequency of the shaker and also the frequency of the response of the bridge at that point in time. With this algorithm in place, the excitation frequencies were determined to the nearest one hundredth of a Hz.

Filtering

The bridge response signals contained some noise as seen in Figure 2. The removal of this noise from the response signals was accomplished by passing the signals through a digital filter. Because the response signals do not go above 11 Hz and the power spectral density's of the signals indicate that the noise is of higher frequency content, a low-pass filter was used to eliminate the noise.

When filtering a signal the noise needs to be removed while maintaining the integrity of the signal. This means that the amplitude of the signal cannot be changed and there should be no phase shift occurring in the signal as a result of the filter. Great care was taken to design a filter that met these criteria.

Different types of filters were experimented with and studied to see which one filtered the signal the best without altering the underlying signal. Chebechev, Butterworth, Kaiser, and many other types of filters and filter windows were examined for their effectiveness. It was determined that a Finite-length Impulse Response (FIR) filter using Blackman's pass window with specific cutoff frequency ranges worked best for the given situation [7].

Digital filters inherently cause a phase shift to occur in the filtered signal. Since the phase angles are extremely important to this project for calculating mode shapes and modal dampings, any phase shift was not desirable. However, the algorithm used to filter the signal permitted the filter to be applied to the data twice. The first filtering of the signal shifts the signal forward and the second filtering of the signal shifts it backwards, thus giving a net phase shift of zero degrees.

Response Signal

With the response signals filtered it became possible to determine their amplitude. The amplitude of the bridge response was found by picking the local minima and maxima from the sinusoidal response curves and their corresponding times out of the signals. By averaging the maximum values and subtracting the average of the minimum values and dividing by 2, the amplitude of the response signal was determined.

When picking the minima and maxima there is a concern that correct values are extracted from the signals. In each instance the time values at the minima and maxima were also used to calculate the frequency of the response signal. If the frequencies extracted from the signal using this method matched the excitation frequencies determined from the shaker, then that was an indication that the correct minima and maxima were extracted from the response signal and that the corresponding amplitude was also correct.

Phase

The phase angle is an indicator of the lag in response between the shaker excitation and the maximum/minimum bridge response. By determining the lag that the response of the bridge had to the excitation of the shaker, the natural frequencies of the bridge can be confirmed and the damping can be calculated at those natural frequencies. At resonance, the lag of the bridge response behind the excitation of the shaker will be either 90 or 270 degrees. With this being the case, the relative phase angles can also be used to help define the mode shapes at the natural frequencies.

The phase difference between the excitation and response signals was found using a method similar to the method used to find amplitude. The maximum points of the shaker signal and the maximum points of the response signals were used to calculate the phase angle difference. This method, however, had some drawbacks in that the accuracy of the phase lag is very sensitive to the data sampling rate. Although the calculated phase angles weren't exact in their values they were defined well enough to verify natural frequencies and mode shapes. However, they were not accurate enough to be used in modal damping calculations.

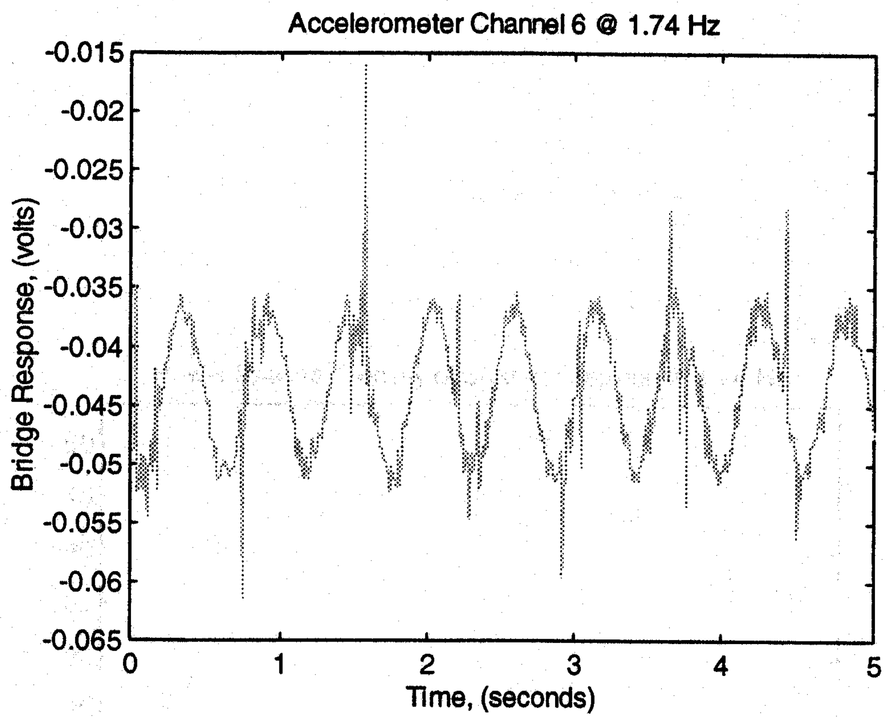


Figure 2. Typical Accelerometer Record

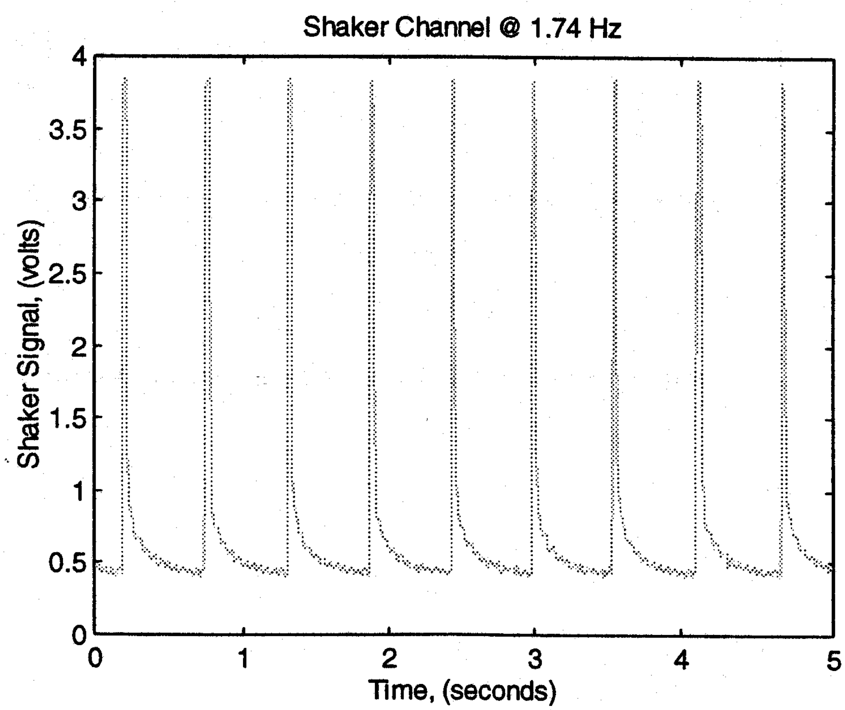


Figure 3. Sample Counter Signal for Mass Shaker

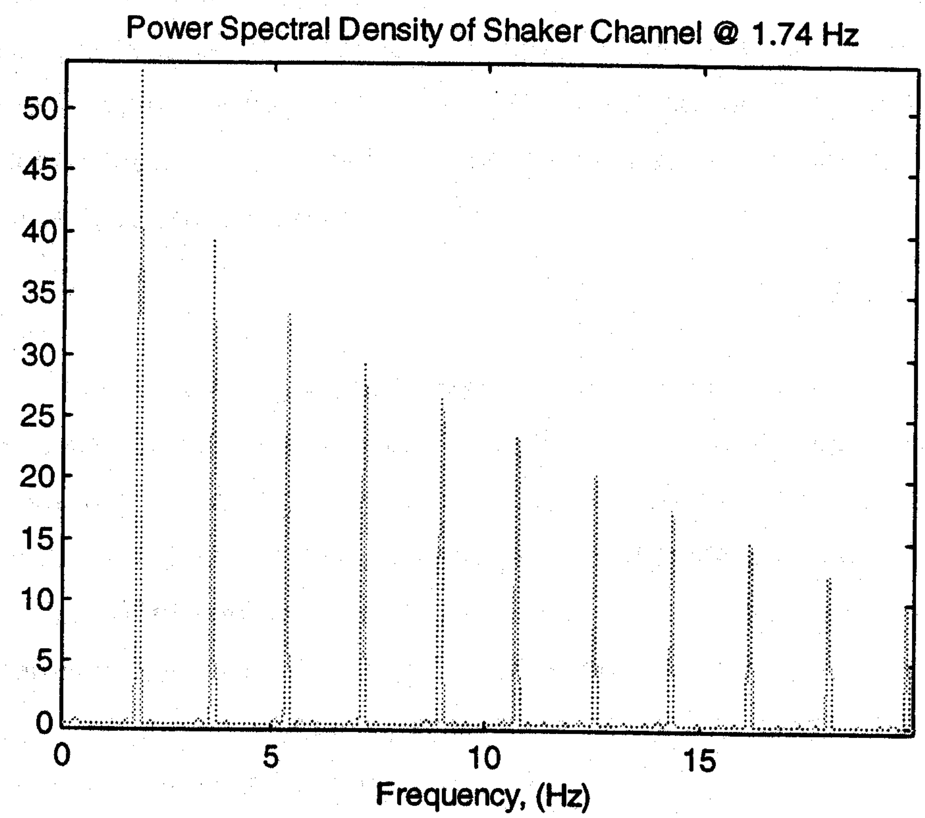


Figure 4. Power Spectral Density for Excitation Signal

IV. Test Results – Nine-Span Bridge Test

Natural Frequencies

The natural frequencies of the bridge can be determined by examining the normalized displacement vs. frequency graphs for the bridge response. A natural frequency is seen on such a curve as a spike. Resonance curves were created and studied in order to determine the natural frequencies of the bridge. A sample resonance curve for the center of span 6 is shown in Figure 5. This curve is typical of the curves developed from the data collected on the other 8 spans of the bridge. Table 1 shows the natural frequencies of five of the lower frequency response modes.

Modes

The phase angles and displacements of the bridge were used to define the mode shapes, which occur at the bridge's natural frequencies. The phase angles were used to determine whether the displacements that were recorded on various spans of the bridge were positive (easterly) or negative (westerly) in direction. Once this was done plots were made of the bridge displacements to determine the mode shapes. The mode shapes and normalized displacements due to the forced vibration excitation for five of the lower frequency response modes are shown in Figures 6a-e.

Because this is a multi-degree of freedom system it is possible that components of modes other than the transverse modes may appear. Due to this fact some spans have spikes in their resonance curves that do not correspond to the frequencies of the transverse modes. This would be especially true for this bridge structure due to the skew. The mode at 2.19 Hz shown in Figure 6b appears as if it were the first transverse mode when in fact it could be a coupled mode between a transverse/longitudinal mode or a transverse/torsional mode. Due to the many degrees of freedom pertaining to this structure it was difficult to assign all of the visible spikes in the resonance curves to a specific mode.

Modal Damping

Phase angles could not be used for modal damping calculations for two reasons. First, the many degrees of freedom present in the bridge made the phase angle vs. frequency plots difficult to use and, second, it ends up that the sampling rate of 100 Hz did not provide sufficient accuracy in the phase angle calculations.

In place of using the phase angles, the Half-Power (Band-Width) Method was more suitable to this situation because it uses the response amplitude vs. frequency plot to calculate damping [12]. The modal damping values calculated for the structure using this method are shown in Table 1. The differences in modal damping at each of the different frequencies is due to the varying response of different structural elements (e.g. boundary conditions) at different frequencies.

Modal Frequency	% Modal Damping
1.79 Hz	3.47
2.19 Hz	2.83
2.53 Hz	4.24
4.53 Hz	4.32
7.26 Hz	3.17

Table 1. Modal Frequencies and Damping

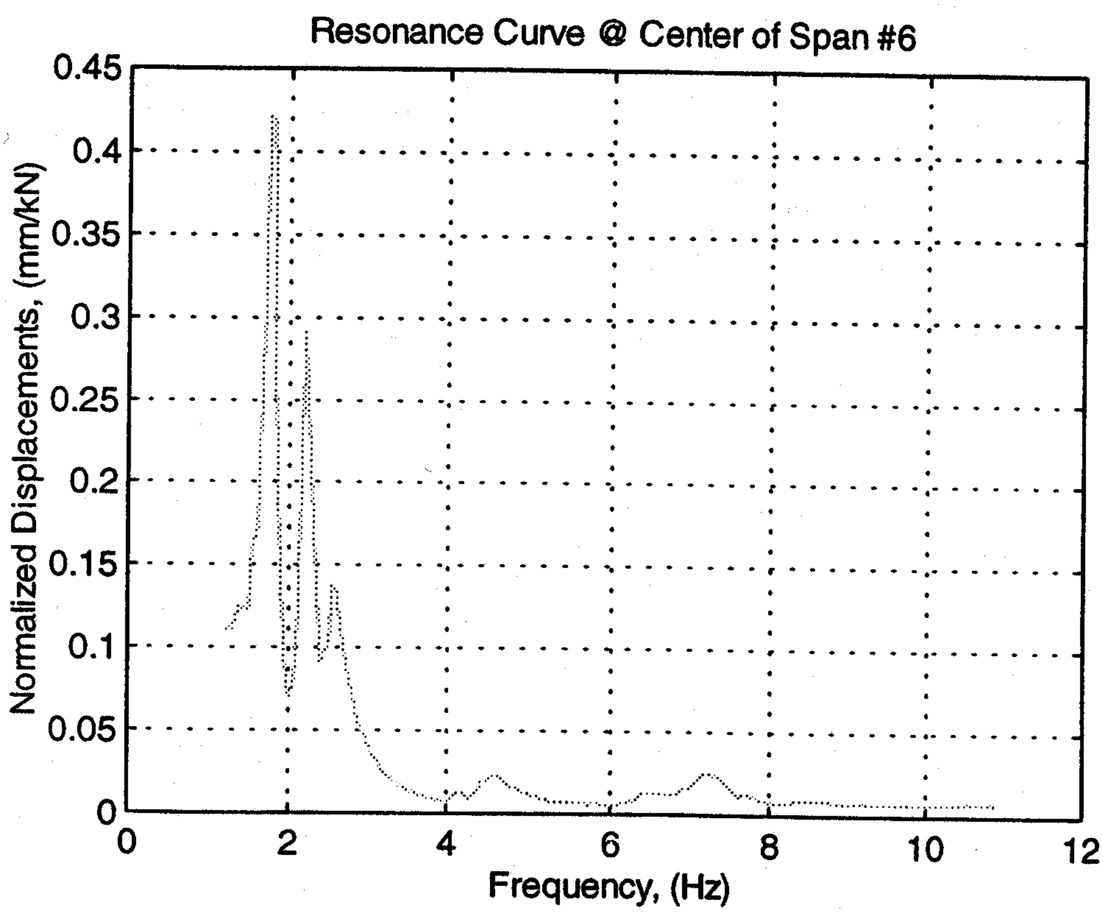
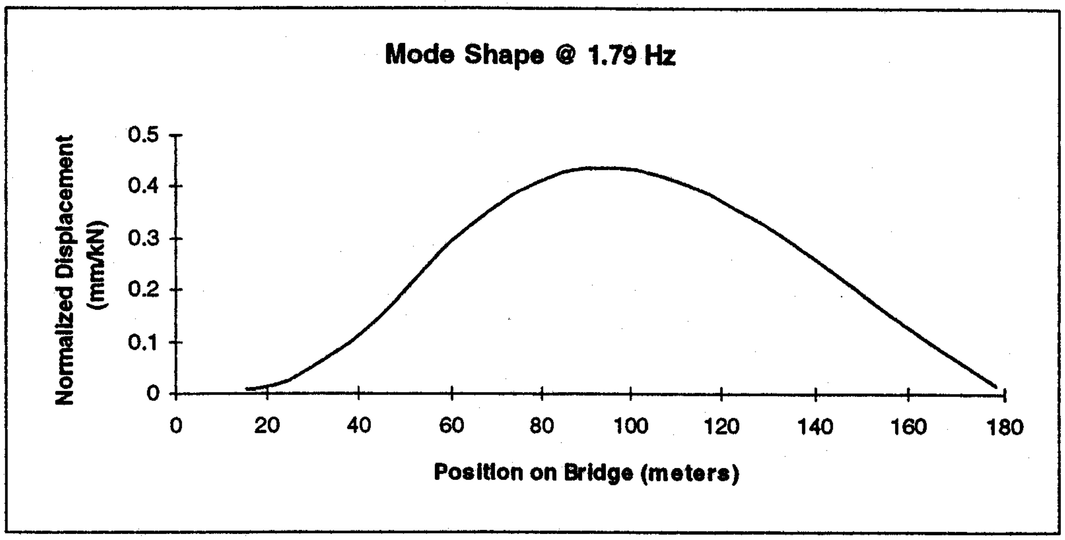
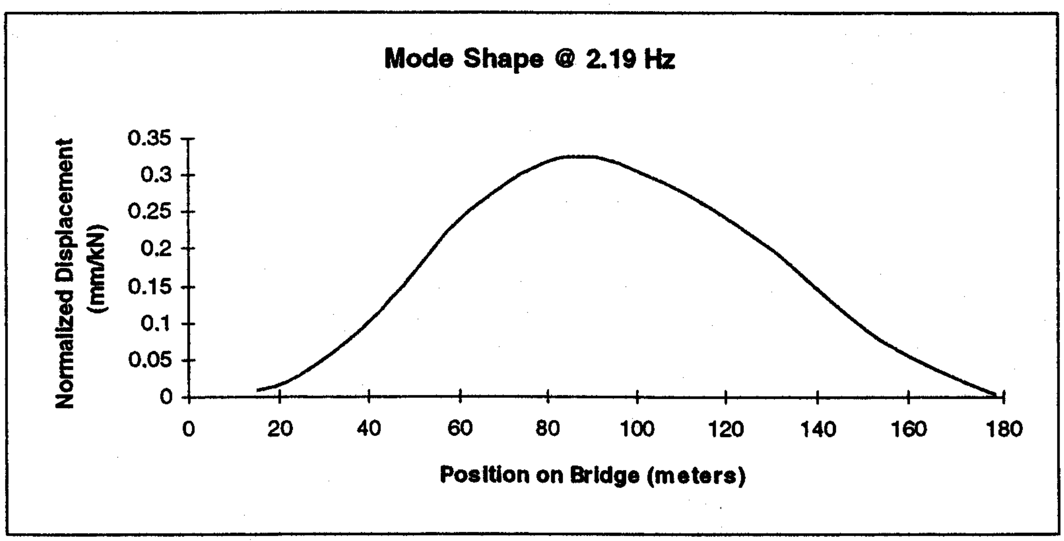


Figure 5. Power Spectral Density for Bridge Response Signal

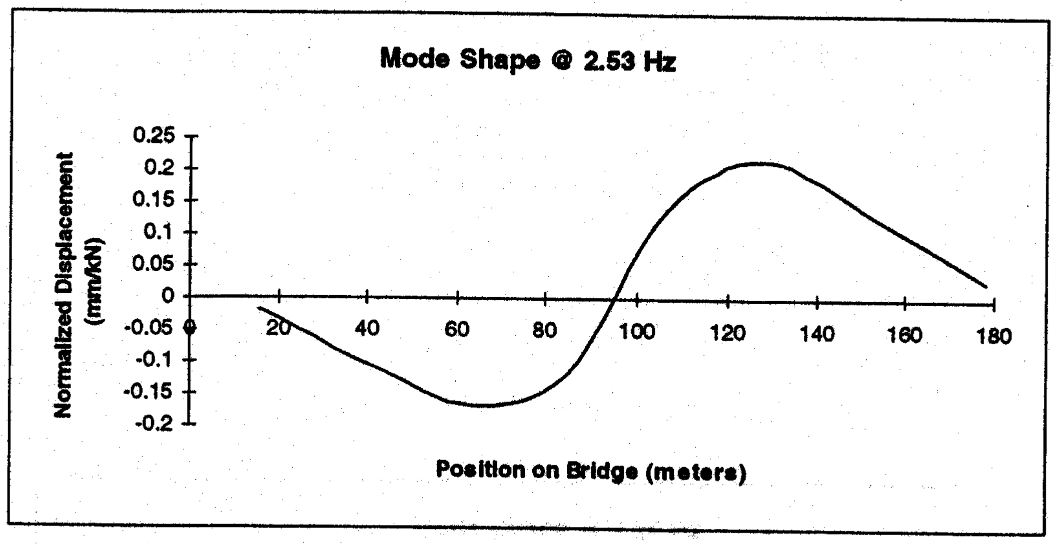


(a)



(b)

Figure 6. Nine-Span Bridge Mode Shapes



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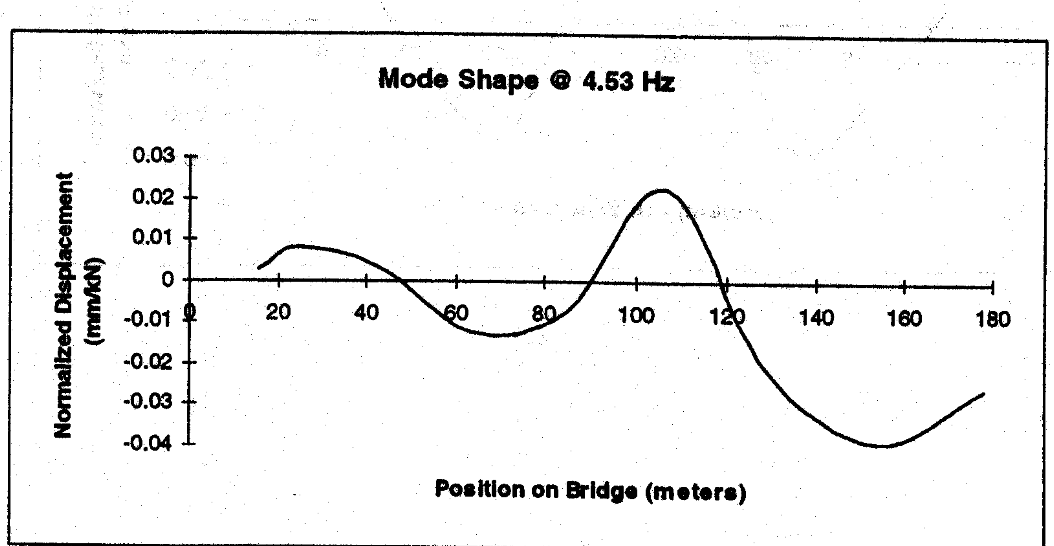
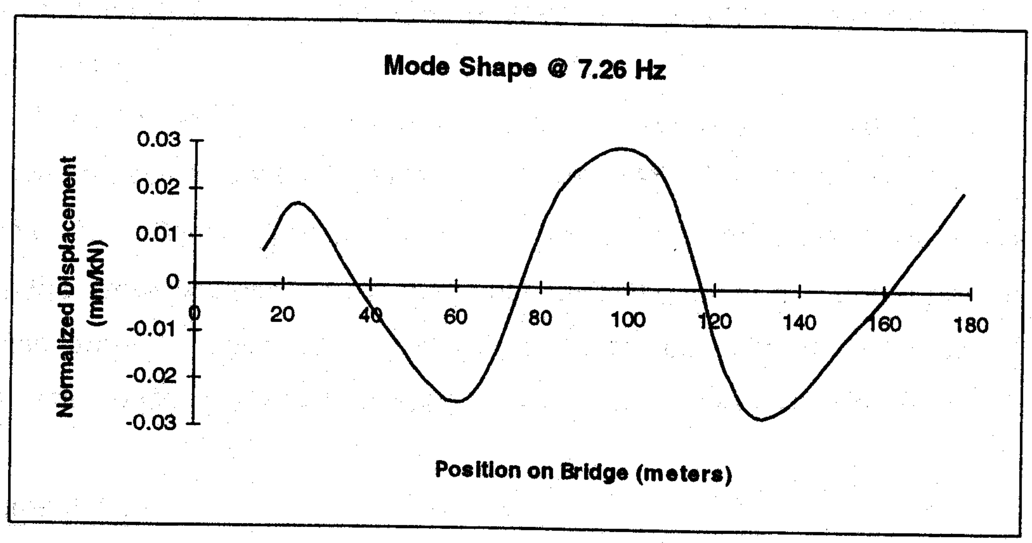


(d)

Figure 6. Nine-Span Bridge Mode Shapes (cont.)



(e)

Figure 6. Nine-Span Bridge Mode Shapes (cont.)

V. Testing of Simple Span

The simple bridge span tested was originally part of the nine-span, northbound overpass structure of I-15 over South Temple Street in Salt Lake City. The original bridge was demolished down to a single, simple span and left so that research could be performed, eventually this simple span was removed for the construction of the new bridge.

The remaining span was span #6, as shown in Figure 1. This span consisted of concrete decking over eight steel girders spanning between the bents on each end of the span. The girders were simply supported using steel rocker bearings.

The 21.87 meter (71.76 feet) span was skewed at 17.5º. The sizes of the bent cap and columns were 1200 mm $\times$ 900 mm (48 $\times$ 36 inches) and 900 mm $\times$ 900 mm (36 $\times$36 inches) respectively. Carbon fiber composites, used to retrofit the bridge, were wrapped around the periphery of the lower and upper part of the columns, around the bent caps by the joints with the columns, and crossing these joints. The configuration of the composite wrapping is shown in Figure 7.

Instrumentation

The mass shaker used to excite the span and the data acquisition equipment used for collecting data were the same as that used for the testing of the entire nine-span bridge. The mass shaker was not moved from its position for the testing of the entire bridge either.

However, the accelerometers were placed on the simple span in a significantly different array. The placement of the ten accelerometers is shown in Figure 7. The dot and arrow of each symbol represent the position and positive direction of each accelerometer. Accelerometers 1,2,3 and 4 were placed at the middle of the perimeter of each side of the deck with accelerometer 5 being placed in the center of the deck. Accelerometers 6 and 7 were placed on the south and north bents, respectively. In an effort to record uplift of the foundation, accelerometers 8 and 9 were placed on the east and west pile cap and number 10 was placed as a free field instrument.

The forced vibration testing of the simple span was performed at seven different times between May and July of 1998. The testing procedures and instrument placement were identical for all seven tests. The sinusoidal forcing was applied in both the North-South and East-West directions at frequencies between 0.5 and 5.5 Hz. This frequency range was based on a finite element analysis of the simple span which indicated that the first three natural frequencies of the structure occurred at less than 3 Hz. The frequency sweep was incremented at steps of 0.05 Hz from 0.5 Hz to 3.5 Hz, above this the step size was 0.25 Hz up to 5.5 Hz. The sampling rate was 200 Hz for all the testing.

The seven different tests were performed in conjunction with destructive lateral load testing and composite repair of both the north and south bents by researchers from the University of Utah [6]. This work provided an excellent opportunity to investigate the potential of system identification as a non-destructive evaluation tool for the detecting structural damage. The condition of the single span structure for each of the tests is outlined below.

<u>Test #1</u>

The north bent had composite fiber wraps placed at the top and bottom of the columns and on the bent cap at the column-bent cap joint. Concrete lateral grade beam struts had been placed between the pile caps at both the north and south bents. The rocker bearings supporting the steel girders on the north bent were replaced with roller bearings that allowed movement of the girders parallel to the bent but not transverse to the bent. This was done to prevent the girders from falling off the bent during the destructive lateral load testing.

<u>Test #2</u>

The north bent was unaltered after the first test. The south bent had been displaced in a direction parallel to the bent (transverse to the bridge centerline) until some yielding occurred in the top joints of the bent and columns. Some minor cracking appeared in the concrete with limited spalling of the concrete cover.

Test #3

The north bent remained unaltered. The south bent was repaired by injecting epoxy into the smaller cracks. Any loose concrete was also removed from the bent.

Test #4

The north bent was displaced laterally to the east until significant yielding occurred in the top of the bent-column joints. Some cracks appeared on the pile cap of the north bent at the pile cap-column joint. The carbon fiber composite de-bonded from the concrete on all the joints. The large displacements caused some crushing of the concrete at the beam-column joints. The south bent was not damaged by the north bent displacement, however, some concrete was chipped away in preparation for the fifth test.

Test #5

The north bent was remained unchanged from the fourth test. The south bent was repaired by shotcreting the exterior of the beam-column joints and then carbon fiber wraps were applied to the joints (see Figure 7).

Test #6

The north bent continued to be unaltered. The south bent was displaced again to the point of significant yielding. More extensive cracking appeared in the beam-column joints. Significant cracking also appeared in the pile cap at the pile cap-column joints. The composite de-bonded from the concrete which was severely damaged underneath the composite.

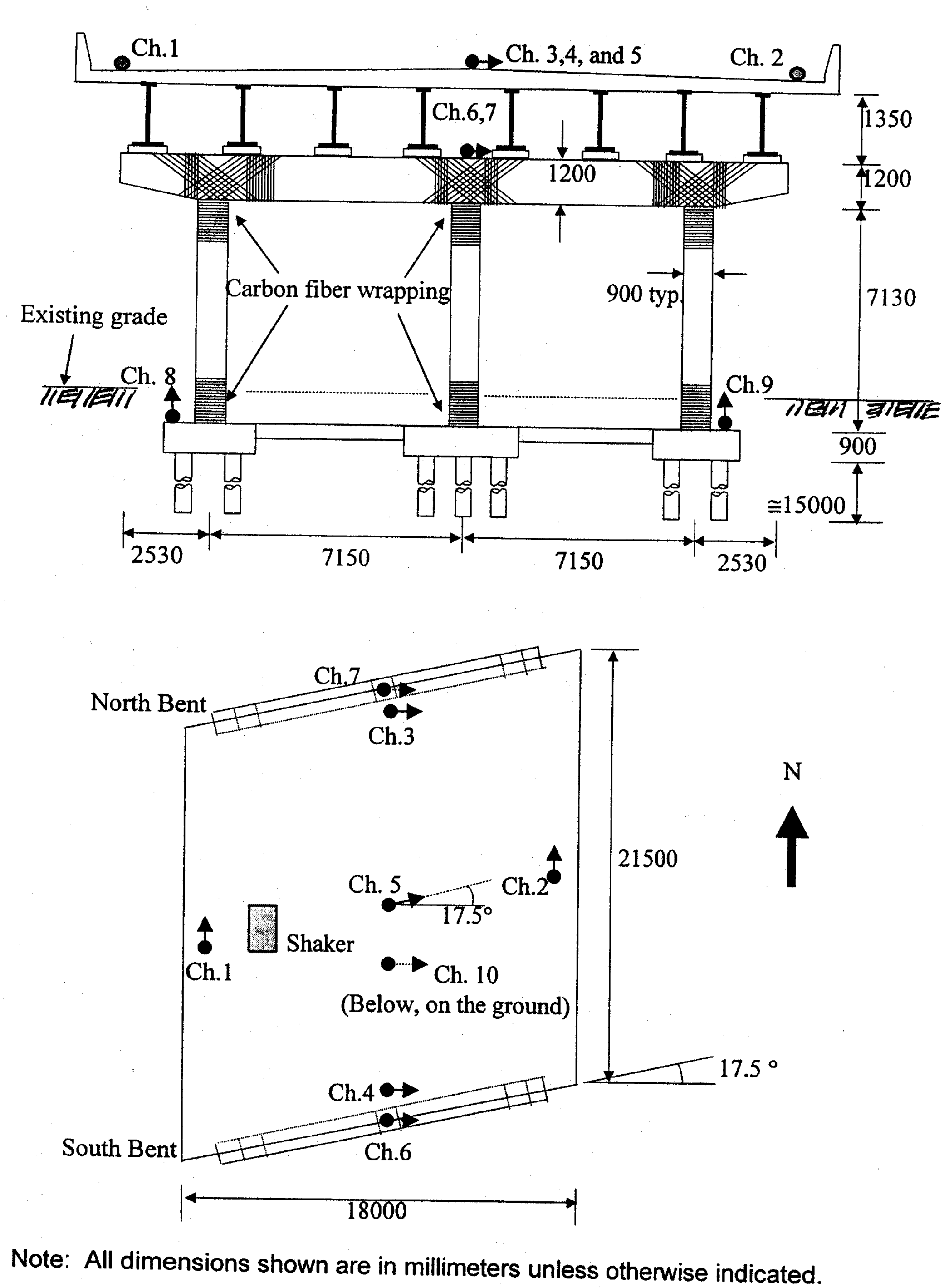
Test #7

The bents were untouched after the sixth test. The roller bearings on the north bent were replaced with the original rocker bearings.

Table 2 summarizes the condition of the simple span during the forced vibration testing and provides the dates for each of the forced vibration tests.

Test	Date	North Bent	South Bent
1	5/14/98	Retrofitted with carbon fiber Rocker bearings were replaced with transverse rollers	Original condition
2	5/26/98	Unaltered since test 1	Laterally displaced until some yielding occurred
3	6/5/98	Unaltered since test 1	Retrofitted with epoxy injection into cracks
4	6/8/98	Laterally displaced until significant yielding occurred	Unaltered since test 3
5	6/25/98	Unaltered since test 4	Retrofitted with shotcreting and carbon fiber
6	6/30/98	Unaltered since test 4	Laterally displaced until moderate yielding occurred
7	7/2/98	Unaltered since test 4 Transverse rollers were replaced with rocker bearings	Unaltered since test 6

Table 2. Condition of Simple Span



Note: All dimensions shown are in millimeters unless otherwise indicated.

Figure 7. Configuration of Simple Span Test Specimen

VI. Test Results – Simple Span

The data from these tests were analyzed in a manner similar to that used to analyze the data from the complete nine-span bridge structure tests.

Natural Frequencies

Graphs for the normalized displacement (by force) versus natural frequency are shown in Figures 8 (a) and (b). The graphs are shown for accelerometer channels 1 and 3 because these two channels show quite well the spikes that indicate the first natural frequencies of the simple span (with the deck acting as a rigid diaphragm) structure. It can be seen from these graphs, through the drop in the natural frequencies, that significant softening (loss of stiffness) of the structure occurred between the first and sixth forced vibration tests. A summary of the change in modal frequencies between each of the forced vibration tests is shown in Table 3 and Figure 9.

The changes in the stiffness characteristics of the simple span structure are shown quite well in Figure 9. The decreases in the first three natural frequencies are shown after damage has been inflicted on the structure. Then when the structure is repaired (after tests 2 and 4) some stiffness is restored to the structure and the natural frequencies increase. These changes are most notable in the higher frequencies.

Modes

The mode shapes can be determined, and natural frequencies verified, by investigating the phase lag. Graphs illustrating the phase lag are shown in Figures 8 (c) and (d). As the structure passes through a natural frequency the phase angle shifts from 0º to 180º (or 180º to 360º) these shifts on the phase angle diagrams can be seen to correspond with the spikes (natural frequencies) shown on the normalized displacement and frequency curves. Therefore, the phase diagrams confirm the natural frequencies of the simple span structure.

The direction of motion for each accelerometer is determined by looking at the phase angles at resonance frequencies. If at a natural (resonant) frequency the phase angle for the motion of an accelerometer is 90º, then the direction of motion of the accelerometer is the same as the excitation force. If the phase angle is 270º, the motion of the accelerometer is opposite that of the excitation force. From this it can be determined which direction the accelerometers are moving in relation to each other and from the mode shapes can be determined.

The first three modes for the simple span structure are rigid diaphragm modes, where the bridge deck moves in a horizontal plane. The first mode is indicated by accelerometers 1 and 2 moving in phase with the excitation and accelerometers 3 and 4 out of phase with the excitation and moving in the opposite direction of accelerometers 1 and 2. The second mode is indicated by accelerometers 1, 2, 3 and 4 all moving in the same direction (in phase). The third mode, a torsional mode, is shown by accelerometers 1 and 3 moving in phase with the excitation and accelerometers 2 and 4 moving out of phase, in the direction opposite to accelerometers 1 and 3. The first three mode shapes, showing the motion of the rigid deck are shown in Figure 10. The mode shapes determined by a finite element analysis, showing the motion of the entire simple span structure, are shown in Figure 11. The natural frequencies, calculated by the finite element analysis, associated with each of these modes are 1.21, 2.32 and 2.71 Hz, respectively.

Test	Mode 1	Fraction of Test 1 Frequencies	Mode 2	Fraction of Test 1 Frequencies	Mode 3	Fraction of Test 1 Frequencies
1	1.30	1.00	2.15	1.00	2.80	1.00
2	1.10	0.85	1.55	0.72	2.10	0.75
3	1.15	0.89	1.65	0.77	2.20	0.79
4	0.85	0.65	1.20	0.56	1.75	0.63
5	0.95	0.73	1.30	0.61	2.10	0.75
6	0.80	0.62	1.10	0.51	1.40	0.50
7	0.80	0.62	1.10	0.51	1.35	0.48

Table 3. Variation of Natural Frequencies with Span Condition

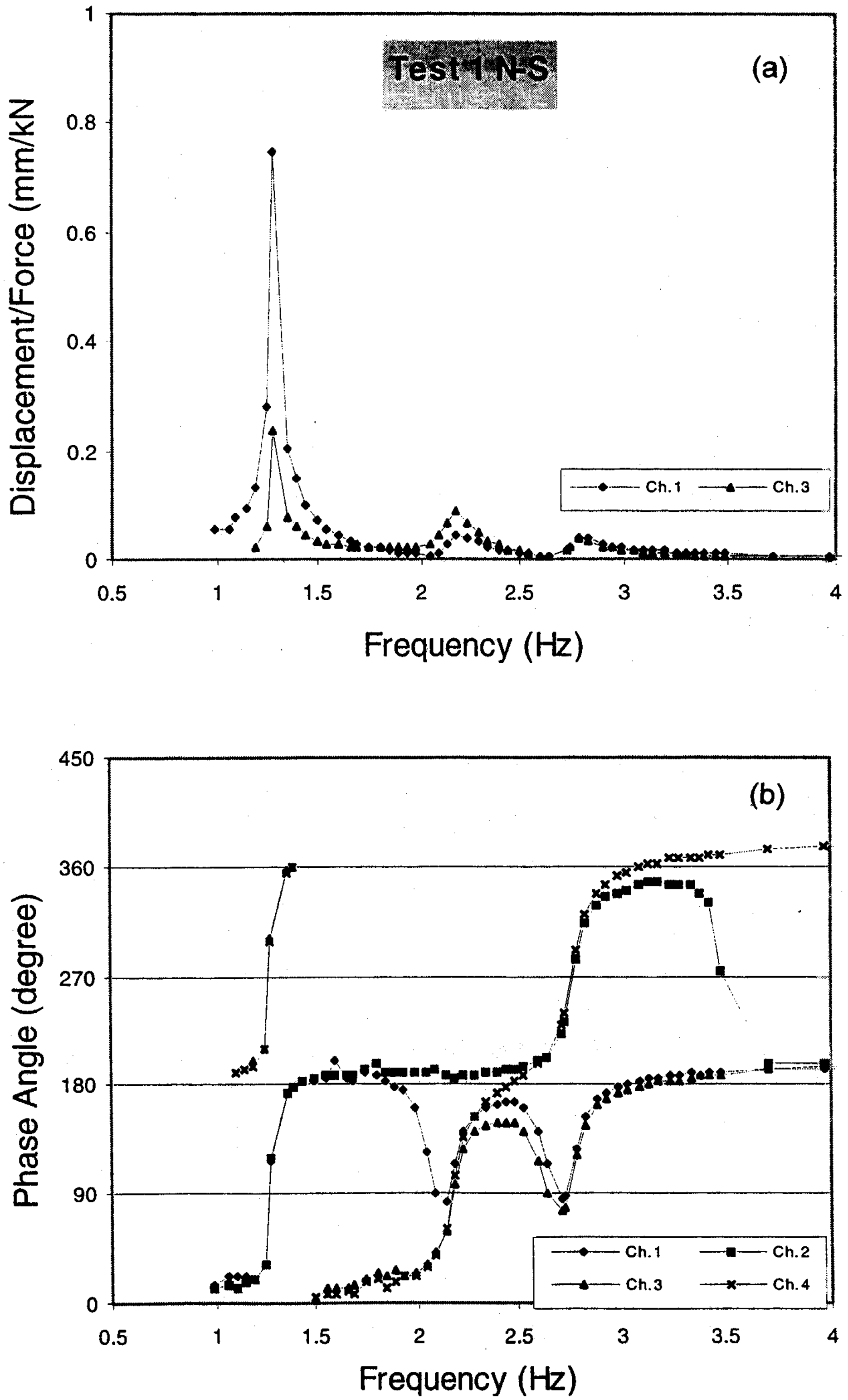


Figure 8. Displacement vs. Natural Frequency and Phase Angles for Simple Span

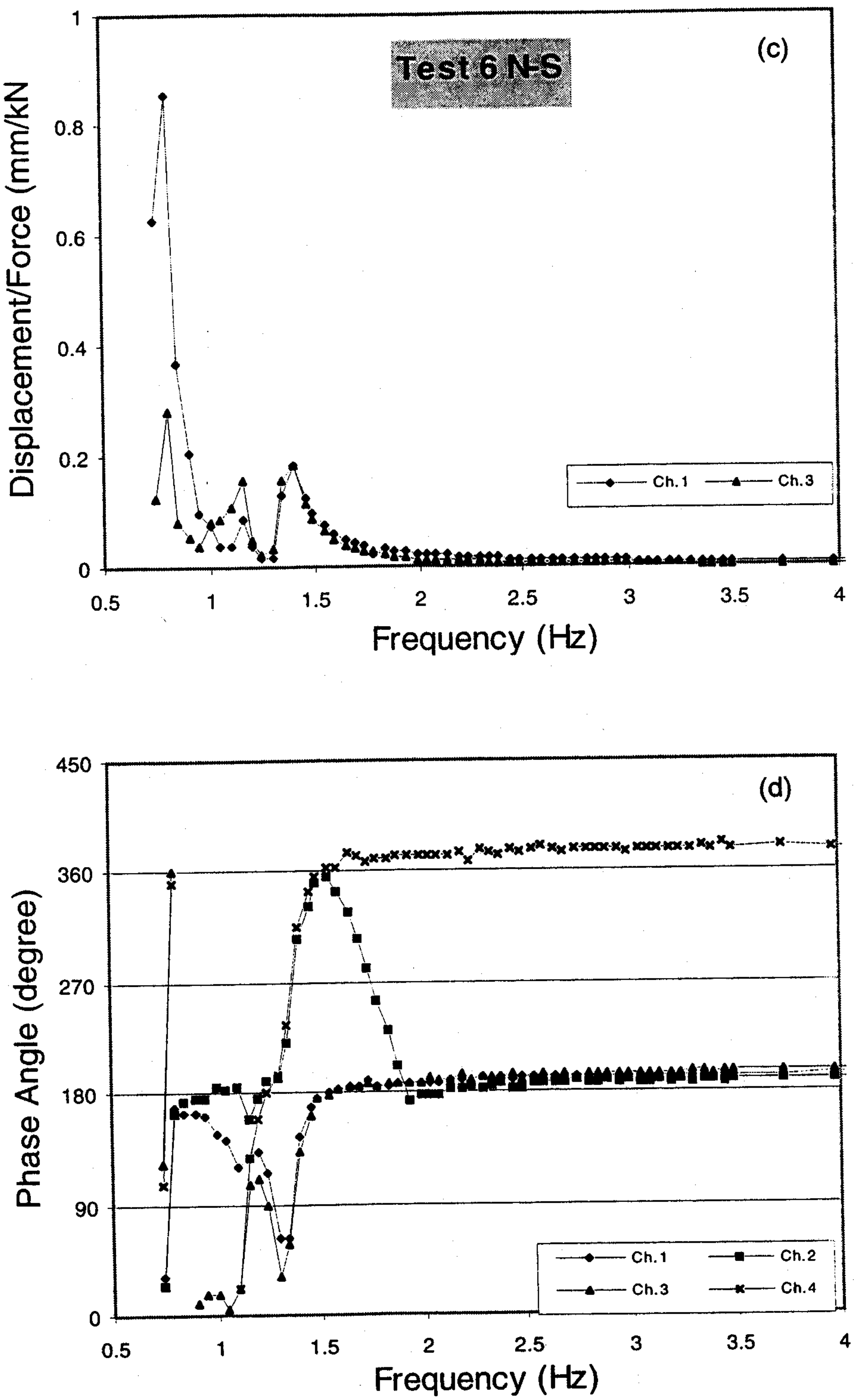


Figure 8. Displacement vs. Natural Frequency and Phase Angles for Simple Span (cont.)

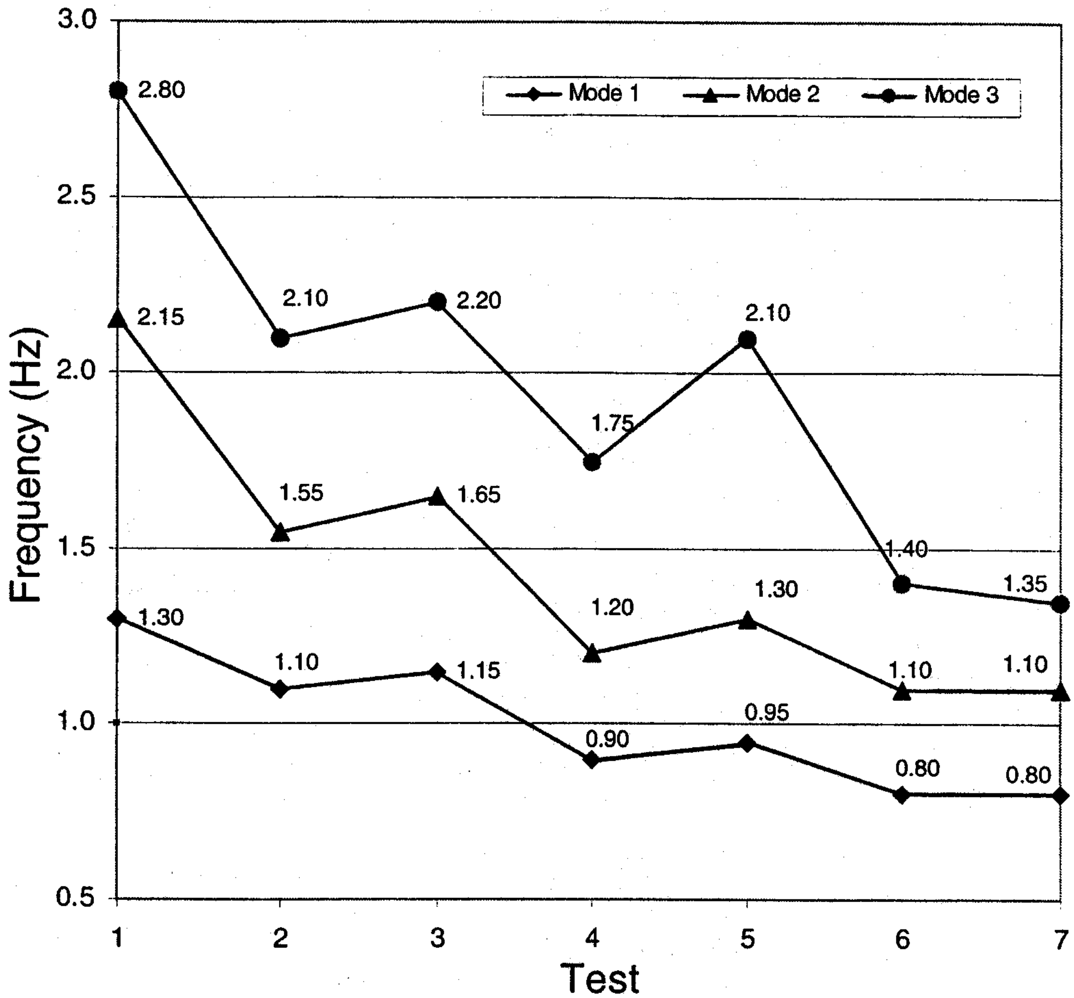


Figure 9. Natural Frequencies for Simple Span

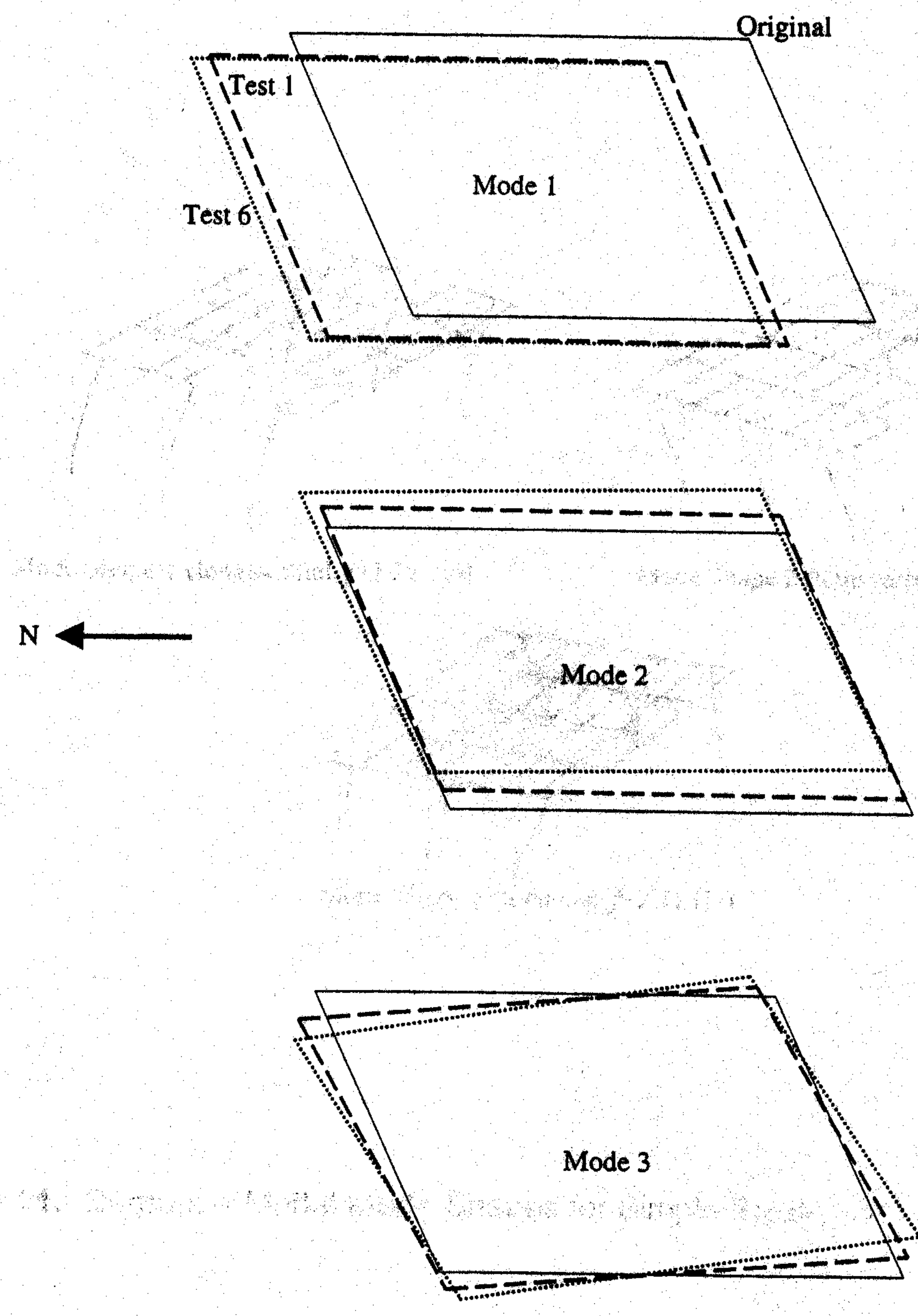


Figure 10. Mode Shapes for Simple Span

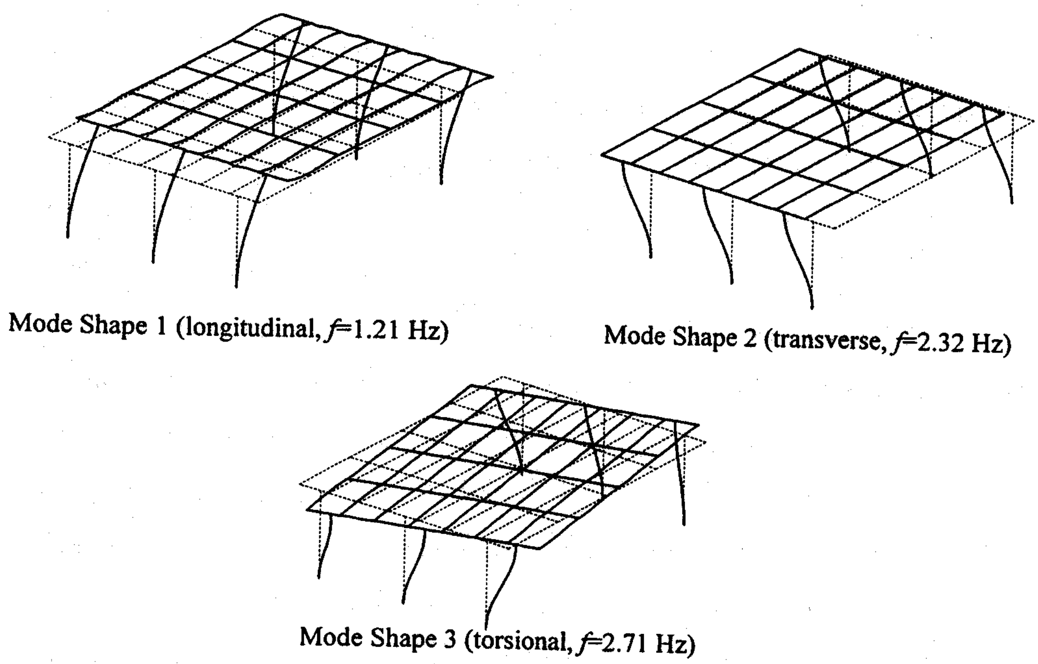


Figure 11. Computer Model Mode Shapes for Simple Span

VII. Conclusions and Recommendations

Conclusions

1. The performance of system identification on large, MDOF bridge structures is possible, using the method outlined in this study.
2. The use of system identification shows promise as a non-destructive evaluation tool for the detection of structural damage.
3. The accuracy of mode shape and phase angle calculations is a direct function of the precision with which the phase angles are calculated. The more degrees of freedom in a structure the more difficult it is to determine phase angle. Calculation of phase angle is also very sensitive to the data acquisition sampling rate.
4. It is essential to have enough instrumentation to record the complete motion of a structure. Nine accelerometers for nine spans, with all instruments oriented in the transverse direction, were not capable of picking up the longitudinal motion of the skewed bridge.
5. The end spans of the nine-span bridge were far enough away from the excitation that the accelerometers on these spans had difficulty in picking up the bridge response above the noise.

Recommendations

1. Further research to verify the use of system identification and modal analysis as non-destructive evaluation tools should be conducted. The reconstruction of I-15, with the bridges to be demolished, is the ideal opportunity to conduct such research.
2. Sufficient instrumentation is essential to the success of system identification. Instruments should be place on each bridge span, to monitor motion in all three global directions, and at the boundaries. Computer models of a structure should be developed to aid in determining where instruments should be placed.

3. On large bridge structures the eccentric mass shaker (excitation source) should be moved to multiple locations.
4. The sampling rate for data acquisition should be at least 20 times the highest excitation frequency. On the nine-span structure sampling at 100 Hz was not sufficient, but the 200 Hz sampling rate used on the simple span proved to be satisfactory.

<u>Implementation</u>

This research project was the first step in a verification process that will examine the potential for using (modal analysis) as a non-destructive evaluation technique for post-seismic damage and long term deterioration of bridge structures. The results of this research indicate that there is potential for using structural identification in this way. Research projects currently being conducted on the I-15 corridor and future research over the next 2 years will be used in combination with this research to validate or negate the use of structural identification as a damage detection tool.

Should this concept prove to be successful in damage detection, then a database of bridge dynamic characteristics would need to be developed for existing bridges through a field-testing program. The bridges to be tested for this database should be selected on the basis of importance to lifelines and susceptibility to damage. Every three to five years these bridges would need to be re-tested to update the database and identify any changes in dynamic characteristics of these bridges which might be the result of long-term deterioration.

In the case of an earthquake any bridge that had been previously tested, with dynamic characteristics contained in the database, could be re-tested to verify the bridge's structural integrity or detect any damage that might have occurred.

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